SEISMIC BEHAVIOUR AND STRENGTHENING OF EXISTING REINFORCED CONCRETE STRUCTURES

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Abstract

The seismic vulnerability associated to existing reinforced concrete buildings, constructed until the late 1970’s, in urban areas of many European countries with moderate to high seismic hazard, is of extreme importance. In that period, reinforced concrete buildings were designed and constructed without considering adequately earthquake provisions, constituting therefore a significant source of risk for our cities. Recent major earthquakes around the world have evidenced that this type of existing buildings lacking appropriate seismic resisting characteristics are very vulnerable.

The present paper focus in four important subjects regarding the seismic behaviour of reinforced concrete buildings, first the importance of the experimental studies on full-scale buildings in particular with two full-scale four-storey reinforced concrete frames tested, in these tests two problems were under study the presence on infill masonry panels in the structure and the use of smooth reinforcement bars, that induces a sudden loss of concrete-steel bond is one of the sources of brittle failure in RC elements. For the presence infill masonry panels it’s presented a simplified macro-model that is able of represents the global behaviour of infill masonry panels and its interaction with RC elements and its application to a case study. Regarding the study of RC buildings smooth reinforcement bars it’s presented the experimental campaign ongoing and the first results and conclusions.

Another identified problem in the seismic behaviour of RC buildings is the behaviour of axially loaded reinforced concrete members under biaxial bending moment. It’s presented the actual lack of experimental studies and the nonexistence of simplified models able to represent these effects and the experimental study of RC columns under uniaxial and biaxial bending.

1. INTRODUCTION

In Europe, many structures are potentially seismically vulnerable due to the late introduction of seismic demands into building codes. Therefore, there is an evident need to investigate the seismic behaviour of existing reinforced concrete (RC) buildings, in order to assess their seismic vulnerability and ultimately to design optimum retrofitting solutions.

The most common causes of inadequate response of building to seismic loading are associated with: i) stirrups/hoops, confinement and ductility; ii) bond, anchorage, lap-splices and bond splitting; iii) inadequate shear capacity and failure; iv) inadequate flexural capacity and failure; v) inadequate shear strength of the joints; vi) influence of infill masonry; vii) vertical and horizontal irregularities; viii) higher modes effect; ix) strong-beam weak-column mechanism, and, x) structural deficiencies due to architectural requirements. However, it should be noted that normally structural damages and failures are associated to the combination of several of these factors. The inadequate response of buildings to seismic demands is, normally, associated to a large number of causes, as reported. Advanced numerical analysis models should be adopted in the assessment of existing buildings and in the design of new ones. For the calibration and development of simplified numerical models able to represent accurately the non-linear behaviour it's particularly important the experimental tests on full scale structures. There are unsolved problems in the application of simplified models, namely in the infill masonry panels participation, the biaxial bending behaviour of RC columns, these two subjects important for new and existent structures and the use of smooth reinforcement bars in buildings constructed until the late 1970's.

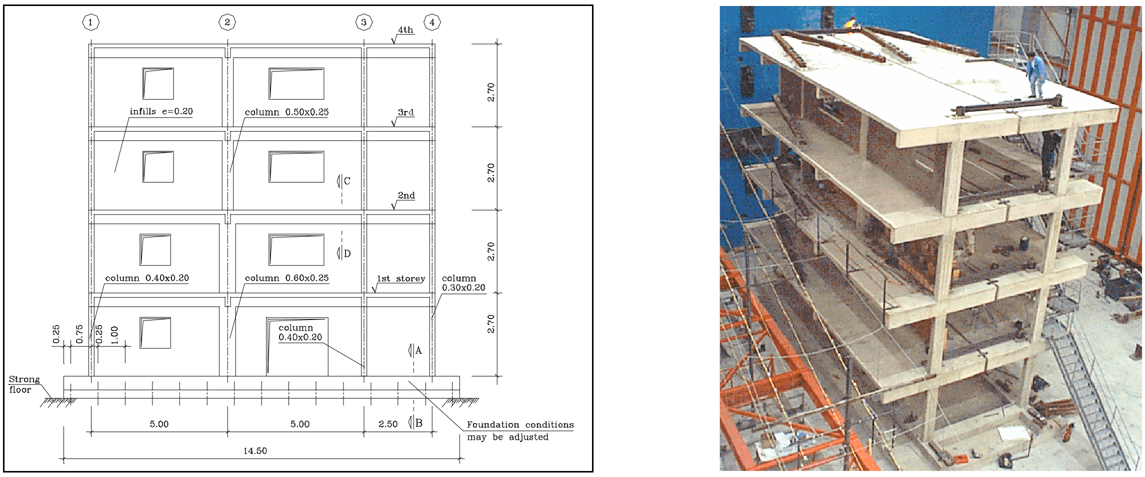
2. TESTS ON A FULL-SCALE STRUCTURE

In the framework of the ICONS Topic 2 - Assessment, Strengthening and Repair - research programme [Pinto *et al.*, 2002], two full-scale four-storey reinforced concrete frames were tested pseudo-dynamically at the ELSA laboratory. The frames, representative of the common practice of design and construction until the late 1970's in most European Mediterranean countries, have been constructed and tested in order to assess the vulnerability of bare and infilled structures and to investigate various retrofitting solutions. This experimental study aimed at assessing the original capacity of existing structures, with and without infill masonry, and to compare the performance of different retrofitting solutions.

The general layout of the building frame model is shown in Figure 1. It is a reinforced concrete 4-storeys full-scale frame with three bays, two of 5m span and one of 2.5m span. The inter-storey height is 2.7m and a 0.15m thick slab of 2m on each side is cast together with the beams. Equal beams (geometry and reinforcement) were considered at all floors. The columns, all but the wider interior one, have equal geometric characteristics along the height of the structure. A comprehensive description of the frames, tests on material samples used in the construction (steel reinforcement and concrete) and PsD test results can be found in [Pinto *et al.*, 2002; Varum, 2003].

The materials considered at the design phase [Carvalho *et al.*, 1999] were a low strength concrete, class C16/20 (Eurocode 2) and smooth reinforcing steel (round smooth bars) of class FeB22k (Italian standards). The reinforcement detailing (lap-splice, stirrup, etc.) adopted is representative of the non-ductile reinforced concrete structures of ~50 years ago. Vertical distributed loads on beams and concentrated loads on the column nodes were considered in order to simulate the dead load other than the self-weight of the frame. These correspond to the following vertical loads: weight of slab 25×0.15=3.75kN/m^2, weight of finishings 0.75kN/m^2, weight of transverse beams 2.5kN/m, weight of masonry infills 1.1kN/m^2 of wall area, and live load 1.0kN/m^2 (quasi-permanent value).

The input seismic motions were defined in order to be representative of a moderate-high European seismic hazard scenario [Campos-Costa and Pinto, 1999]. Hazard consistent acceleration time series (15 seconds duration) were artificially generated yielding a set of uniform hazard response spectra for increasing return periods. Acceleration time histories for 475, 975 and 2000 years return periods (yrp) were used in the tests (PGA of 218, 288 and 373cm/s^2, respectively).

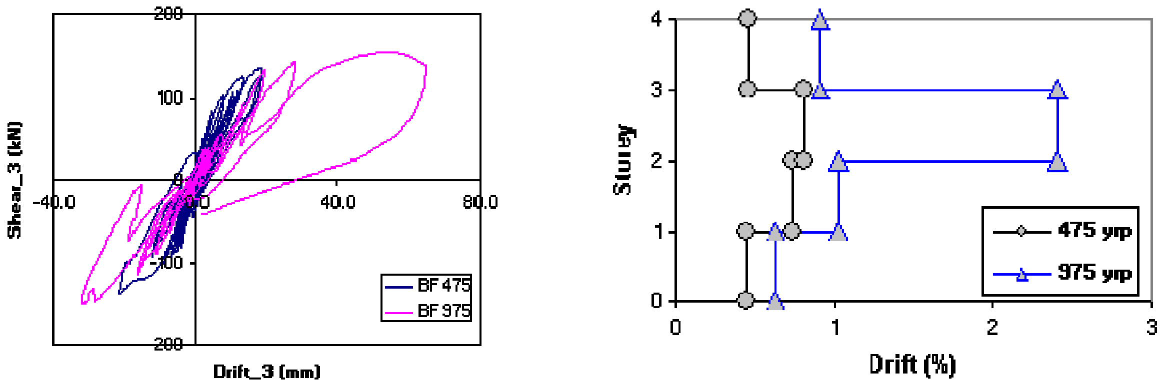


a) b)

Figure 1. Tested frames: a) elevation views of the frames; b) models in the ELSA laboratory

2.1. Test results of the bare frame structure

The bare frame (BF), was subjected to one PsD earthquake test corresponding to 475-yrp and subsequently to a second PsD test carried out with a 975-yrp input motion using pseudo-dynamics testing techniques.



a) b)

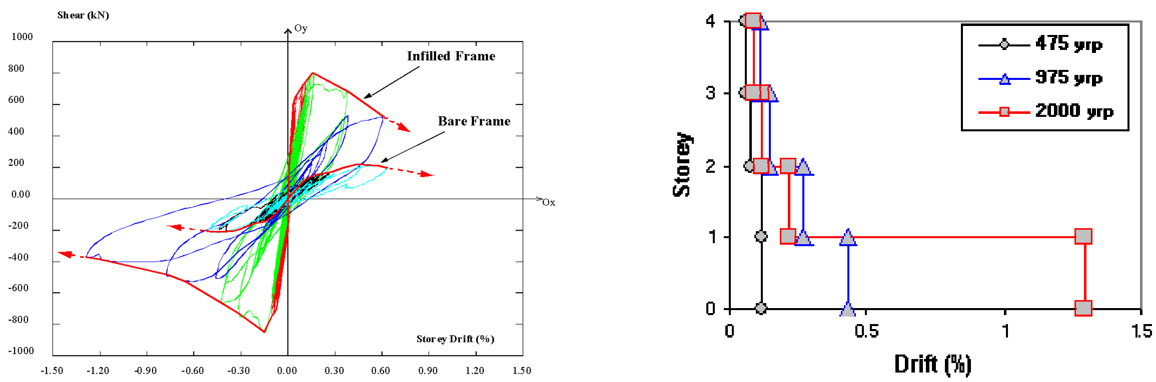
Figure 2. BF test results: a) shear-drift diagram at the 3rd storey; b) maximum inter-storey drift profiles

The 975-yrp test was stopped at 7.5 seconds, because imminent collapse was attained at the 3rd storey. The significant reduction in terms of stiffness and strength from the 2nd to the 3rd storey (vertical irregularity), coupled with the inadequate lap-splicing and shear reinforcement, induced the concentration of larger inter-storey drift demand, and consequently damage, in the 3rd storey, developing a soft-storey mechanism at the 3rd storey. Results from these tests are given in Figure 2, in terms of storey shear versus storey drift at the 3rd storey and maximum inter-storey drift profiles.

2.2. Test results of the infilled structure

Figure 1 shows the general layout of the structure including infill panels and the type and location of the openings. The 150mm thick infill-walls (non-load bearing) were constructed after the reinforced concrete frame. Representative materials and construction techniques were used, namely: Italian hollow clay (ceramic) blocks horizontally perforated, with dimensions: 0.12m thick, 0.245m base-length and 0.245m height. The mortar joints are approximately 1.5cm thick and a 1.5cm thick plaster was applied on both sides of the walls. The same mortar proportioning was used for bed joints and plaster (1:4.5 - hydraulic binder:sand). The infilled frame specimen was subjected to three consecutive PsD earthquake tests corresponding to 475, 975 and 2000-yrp. During the 2000-yrp

PsD test, the masonry infills at the 1st storey collapsed and the test was stopped at ~5 seconds. Results from these tests are given in Figure 3 in terms of storey shear-drift and maximum inter-storey drift profiles. For the 475-yrp test, overall, the infilled frame structure behaved very well. The 975-yrp earthquake caused a significant damage to the infill alls in the bottom storey, with some minor damage to the concrete beam-column joints and columns at this level. Smaller amount of damage in similar locations were noted in the 2nd storey. No significant damage was observed in the upper two stories. It was recognised that the infill frame had become, at the end of the 975-yrp test, a soft-storey infill frame structure. Nevertheless, it was subjected to the 2000-yrp earthquake signal in order to study how gradually the lateral strength dropped off with increasing drift. The storey shear versus drift hysteresis loops clearly illustrate that the load deflection characteristics approach those of the bare frame as the drifts increase to values in excess of 1% (see Figure 3). The infilled frame demonstrated completely different behaviour compared to the bare frame. Infills protect the RC structure but also prompt storey mechanisms and cause shear-out of the external columns in the joint region.



a) b)

Figure 3. IN test results: a) 1st storey shear-drift diagrams and envelope curves (comparison with the BF); b) maximum inter-storey drift profiles

3. INFILL MASONRY PANELS

The vast majority of buildings, in earthquake prone areas in Europe, constructed before the 1980's are seismic deficient in terms of our current understanding. In fact, in some European countries until the 1960's no specific seismic design provisions were included in building codes and, from that period on, only seismic equivalent lateral loading were considered in building design.

It is inadequate to assume that masonry infill panels are always beneficial in terms of structural response. The contributions of infills to the building's seismic response can be positive or negative, depending on a series of phenomena and parameters such as, for example, relative stiffness and strength between the frames and the masonry walls. In recent earthquakes, numerous buildings were severely damaged or even collapsed as a result of the structural modifications to the basic structural system induced by the non-structural masonry partitions. Masonry infill panels can increase substantially the global stiffness of the structure. Consequently, its natural period will decrease and, depending on the seismic spectrum values at the vicinity of the bare structure natural period, the seismic forces can increase.

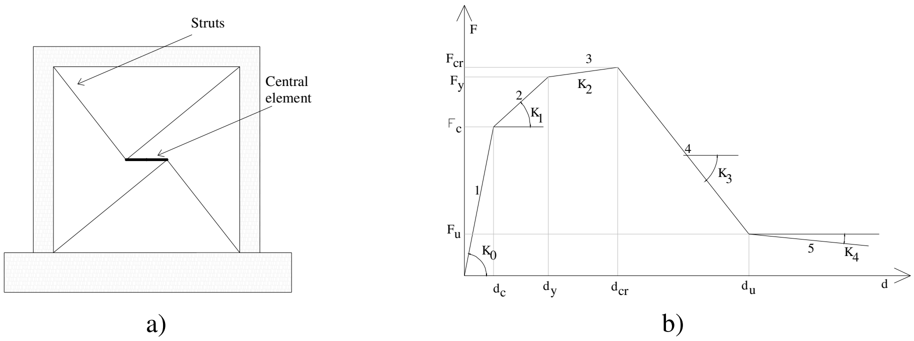
There are many different techniques proposed in the literature for the simulation of the infilled frames, which can be basically divided in two groups, namely the micro-models and the simplified macro-models. The micro-models considers a high level of discretization of the infill masonry panel, in which the panel is divided into numerous elements to take into account the local effects in detail, the simplified models are supported in simplifications with the objective of representing the global behaviour of the infill panel with a few structural elements. Micro-models can simulate the structural behaviour with notable detail, requiring different types of elements to represent respectively the bricks, mortar, interface brick-mortar, interface masonry-frame and the frame elements. However, they are computational intensive and difficult to apply in the analysis of large structures. Relatively to the simplified models, the most commonly used technique to model infill panels is based on a single or multiple compressive equivalent diagonal struts strategy.

3.1. Standards and recommendations

Being aware of the importance of infill masonry elements in the behaviour of RC buildings in the last few years the new codes have included some provisions regarding the consideration of the infills and their influence on the structural response. For example, the European code, Eurocode 8 (EC8) [CEN, 2003], include provisions for the design of infilled RC frames (in its section 2.9). EC8 specifies that the period of a structure to be used to evaluate the seismic base shear shall be the average of that for the bare frame and for the elastic infilled frame. Frame member demands are then determined by modelling the frame structure without the infills. The influence of irregular infill distributions, in plan and in elevation, is addressed in EC8. The American guideline FEMA-273 (2003) and FEMA 356 (1997) includes a procedure to assess the structural response of buildings, considering the infills panels. According to this document, masonry infill panels shall be represented by the equivalent diagonal struts. The struts may be placed concentrically across the diagonals, or eccentrically to directly evaluate the infill effects on the columns. The shear behaviour of masonry infill panels is considered as a deformation controlled action. FEMA-273 provides deformation acceptance criteria. The linear procedure involves a comparison between the design elastic shear force in an infill panel with the factored expected shear strength of the panel.

3.2. Proposed simplified model for masonry infill panels

The macro-model proposed here is an improvement on the commonly used equivalent bi-diagonal-strut model [Rodrigues, 2005]. The proposed model considers the interaction of masonry panel behaviour in both directions; damage to the panel in one direction affects its behaviour in the other direction. Therefore, the proposed model more accurately represents global response and energy dissipation during structural response. In the proposed infill panel model, each masonry panel is structurally defined by considering four support strut-elements, with rigid behaviour, and a central strut element, where the non-linear hysteretic behaviour is concentrated (Figure 4-a). The forces developed in the central element are purely of tensile or compressive nature.



a) b)

Figure 4. Infill masonry model proposed: a) macro-model; b) force-displacement monotonic behaviour curve

The non-linear behaviour is characterized by a multi-linear envelope curve, defined by nine parameters (Figure 4-b), representing: i) cracking (cracking force, F_c; cracking displacement, d_c); ii) yielding (yielding force, F_y; yielding displacement, d_y); iii) maximum strength, corresponding to the beginning of crushing (F_{cr}; and corresponding displacement, d_{cr}); iv) residual strength (F_u) and corresponding displacement (d_u); the fifth branch of the behaviour curve is defined by its stiffness (K_4). A different behaviour curve can be defined for each loading direction, which allows for the consideration of non-symmetrical behaviour. The hysteretic rules calibrated for masonry models are controlled by three additional parameters, namely: α - stiffness degradation; β - "pinching" effect; and, γ - strength degradation.

The non-linear behaviour of the central element is characterized by the proposed penta-linear monotonic curve and the corresponding hysteretic rules, inspired on the Costa and Costa tri-linear model for RC elements, allowing the determination of the response to cyclic loads as a function of the material's behaviour (defined by the envelope curve and hysteretic parameters). The hysteretic rules are briefly exemplified in Figure 5.

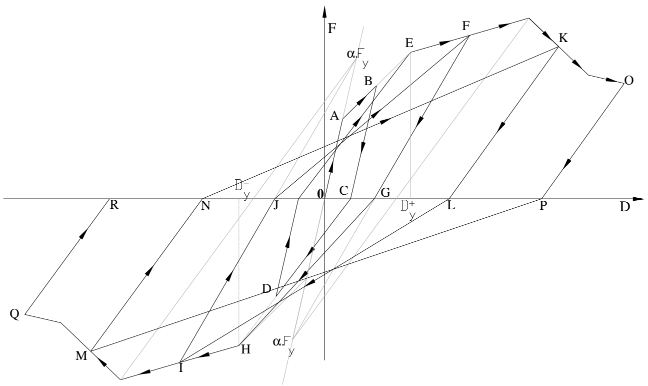


Figure 5. Hysteretic rules for the implemented model

3.3. Application of the proposed infill masonry model

An existing building representative of the Modern Architecture in Portugal was studied. The block plan is rectangular with 11.10m width and 47.40m length. The building has the height of 8 habitation storeys plus the pilotis height at the ground floor.

The building geometry and dimensions of the RC elements and infill walls were given in the original project (fifties), and were confirmed in the technical visits. The building has nine storeys and the structure is mainly composed by twelve plane frames oriented in the transversal direction (direction Y). The building was analysed with a simplified plane model for each direction (X - longitudinal direction, Y - transversal direction).

The twelve transversal plane frames have the same geometric characteristics for all beams and columns. However, three different frame-types (labelled A, B and C) were identified, according to reinforcement detailing differences. A peculiar structural characteristic of the type of buildings, with direct influence in the global structural behaviour, is the ground storey without infill masonry walls. Furthermore, at the ground storey the columns are 5.5m height. All the upper storeys have an inter-storey height of 3.0m.

In the two structural models (directions X and Y), a concrete slab with 1.25m width and 0.20m thick was considered. A detailed definition of the existing infill wall panels was considered in the global structural models, namely, openings dimensions and position in the panel, interface conditions between panel and surrounding RC elements, masonry material's properties. The global behaviour curves for each masonry panel were estimated according to the Zarnic and Gostic (1998) empirical expressions.

3.3.1. Earthquake input signals

Three artificial earthquake input series were adopted for the seismic vulnerability analysis of the building (Figures 6 to 8). The first series (A) was artificially generated for a medium/high seismic risk scenario in Europe, for various return periods (Table 1). The second and third series (B and C, respectively) were generated with a finite fault model for the simulation of a probable earthquake in the South of Portugal, calibrated with real seismic actions measured in the region. The earthquakes of the B and C series were scaled to the peak ground acceleration of series A, for each return period. In Table 1 are presented the peak ground acceleration and the corresponding return period for each earthquake's intensity.

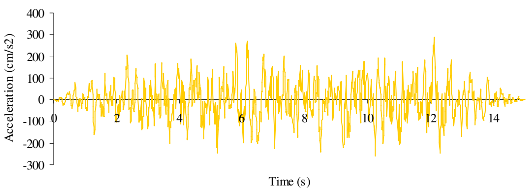


Figure 6. Accelerogram A

TABLE 1: Reference earthquake action (peak ground acceleration and corresponding return period)

Return period (years)	Peak ground acceleration (×g)
73	0.091
475	0.222
975	0.294
2000	0.380
3000	0.435
5000	0.514

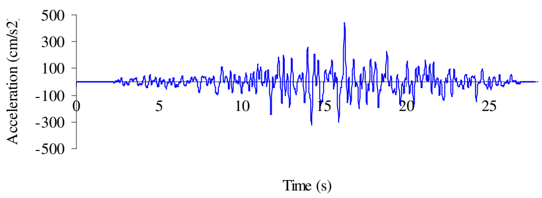


Figure 7. Accelerogram B

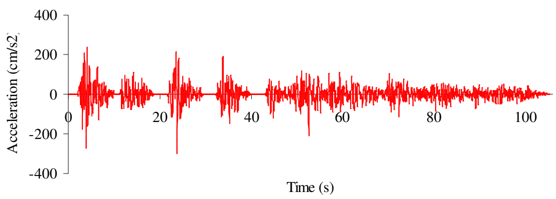


Figure 8. Accelerogram C

3.3.2. Results analysis

In Figures 9 and 10 are illustrated, for the longitudinal and transversal direction respectively, the numerical results in terms of envelop deformed shape, maximum inter-storey drift, and maximum storey shear, for each earthquake input motion of the series A (73, 475, 975, 2000, 3000, 5000 years return period).

From the analysis of the results in terms of building envelop deformed shape and inter-storey drift profile, for both directions, it can be concluded that the deformation demands are concentrated at the first storey level. In fact, the absence of infill masonry walls at the ground storey and the larger storey height (5.60m for the 1st storey and 3.00m for the upper storeys), induces an important vertical structural irregularity, in terms of stiffness and strength.

For all the structural elements (columns and beams), and for all the seismic input action levels, the shear force demand assumes a value inferior to the corresponding shear capacity, which confirms its safety in shear.

From the numerical analyses performed, it was verified for the earthquake input motions that the infill masonry panels basically do not reveals any damage. In fact, due to the building structural system and its behaviour, on one hand, and to the absence of masonry panels at the ground floor, on the other hand, the global deformation demand is concentrated at the ground floor, and, therefore, the majority of the infill masonry panels present a linear behaviour.

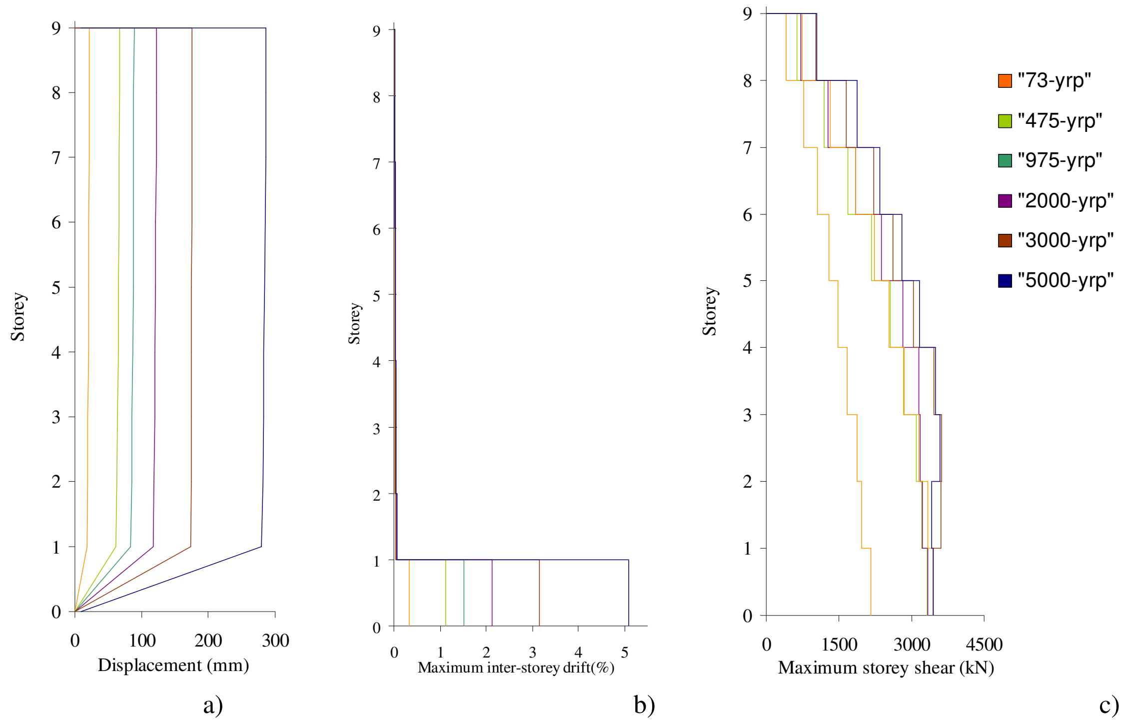


Figure 9. Results for the longitudinal direction (X) and earthquakes of the series A: a) envelop deformed shape; b) maximum inter-storey drift profile; c) maximum storey shear profile

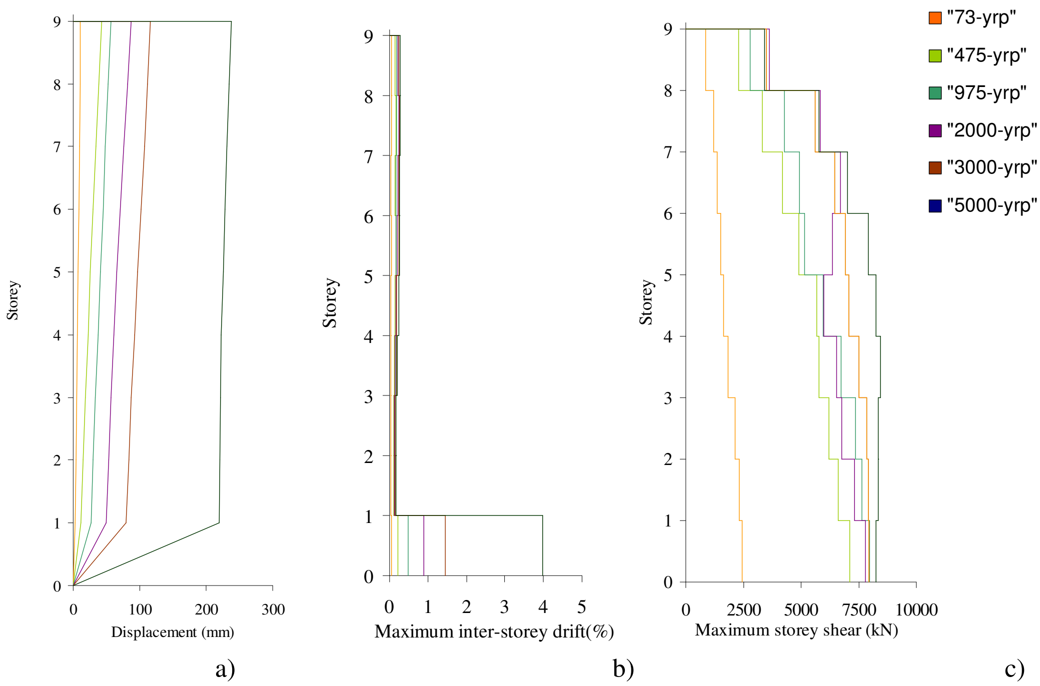


Figure 10. Results for the transversal direction (Y) and earthquakes of the series A: a) envelop deformed shape; b) maximum inter-storey drift profile; c) maximum storey shear profile

3.3.3. Seismic safety assessment of the building

For each direction, the building structure was analysed to three series of earthquakes with increasing intensities, in order to estimate deformation demands, and consequently damage levels.

The VISION-2000 [SEAOC, 1995] document proposes performance objectives for buildings and for three performance levels (called: Basic, Essential Hazardous, and Safety Critical). For the building under study, and due to the nature of its use, the structural safety was investigated for the Basic Performance Objectives proposed at the VISION-2000 (see Table 2, where the performance objectives are marked with an "X").

The obtained results allow verifying the safety according to the hazard levels specified, for example, in VISION-2000 and ATC-40 (1996) documents. In Tables 2 and 3 are presented the acceptable drift limits, for each structural performance level, according to the ATC-40 and in VISION 2000 proposals, respectively.

TABLE 2: Storey drift limits according to the ATC-40 (1996)

	Performance Level			
	Immediate Occupancy	Damage Control	Life Safety	Structural Stability
Drift Limit	1%	1-2%	2%	$0.33\frac{V_i}{P_i} \approx 7\%$

TABLE 3: Storey drift limits according to the VISION-2000 [SEAOC, 1995]

	Performance Level			
	Fully Operational	Operational	Life Safe	Near Collapse
Drift Limit	0.2%	0.5%	1.5%	2.5%

In Figures 11 and 12 are represented the vulnerability functions in terms of maximum 1st storey drift, presented in the previous section, with indication of the safety limits proposed at the ATC-40 and VISION-2000 recommendations.

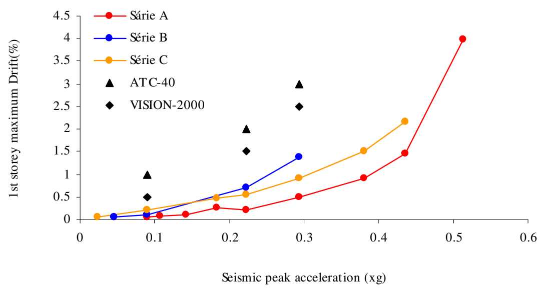


Figure 11. 1st storey drift *vs.* peak acceleration and safety limits (transversal direction - Y)

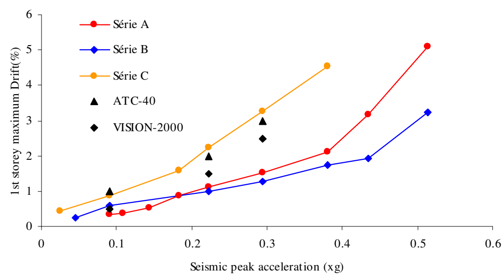


Figure 12. 1st storey drift *vs.* peak acceleration and safety limits (longitudinal direction - X)

Comparing the maximum storey drift demands with the safety limits proposed at the ATC-40 and VISION-2000 recommendations, it can be concluded that the building safety is guaranteed the transversal direction (Y), for the three earthquake input series considered. For the longitudinal direction (X), the safety is guaranteed for series A and B, but not for C series.

3.3.4. Final remarks

The global structural safety of a modern architecture style building in Portugal was investigated. The analysis performed demonstrates the capacities of the models and of the integrated computer program, PORANL, in the representation of the global response and seismic behaviour of RC building structures. Particularly, the non-linear bending behaviour of slender RC elements, and the participation of the infill masonry panels in the global seismic response of buildings was numerically simulated.

Although the first numerical results generally indicate the building safety for the Basic Safety Objectives, according to the international seismic recommendations (ATC-40 and VISION-2000), it should be pointed out that additional analyses have to be performed to validate this first conclusions.

The input motion earthquakes adopted for these analyses can be not fully representative of all the probable earthquake actions in the South of Portugal. Additional analyses should be performed using other earthquake input motions.

The model adopted for these analyses does not consider the geometric non-linearity, which can increase significantly the moments in columns and global storey lateral deformations (drifts). Therefore, to guarantee the seismic safety verification of the building, it is judged focal to verify the results using a model that considers the geometrical non-linearities.

It is evident the high vulnerability of this building structural typology, which exist in a considerable number in Lisbon region. The high seismic risk associated to these buildings can be significantly reduced with the adoption of adequate retrofitting solutions. The seismic retrofitting of these buildings can be performed adopting economic solutions, since intervention can be resumed at the ground storey, usually without infill masonry walls, in this typology, for example with bracing systems and, eventually, combined with energy dissipation devices. After the building assessment and design of a retrofitting solution, its efficiency must be always evaluated.

4. RC ELEMENTS WITH SMOOTH REINFORCEMENT STEEL

A great part of existing RC buildings all over the world were built before the 70's, previously to the introduction of seismic-oriented design. In fact, many were designed to withstand only gravity loads. More, the majority was built with smooth plain reinforcement bars that exhibit poor bond. As a consequence of poor reinforcement details and absence of any capacity design principles, a significant lack of ductility at both the local and global levels is expected for these structures, resulting in inadequate structural performance even under moderate seismic excitation [Pampanin *et al.*, 2000; Hertanto, 2005].

The sudden loss of concrete-steel bond is one of the sources of brittle failure in RC elements, and is reported to have been the cause of severe local damage and even collapse of many structures during earthquakes. Even if no anchorage failure occurs, the hysteretic behaviour of RC structures, namely when subjected to alternate actions (like earthquakes), is highly dependent on the interaction between steel and concrete [CEB, 1996]. Perfect bond is usually assumed in the analyses of RC structures, implying full compatibility between concrete and reinforcement strains. However, this assumption is only valid for early loading stages and low strain levels. As the loads increases, cracking and breaking of bond unavoidably occurs and relative slip between the reinforcement bars and the surrounding concrete (bond-slip) takes place in the structural elements [Varum, 2003]. Different strains are observed in both materials and the stress distribution is affected. RC structural elements built with smooth reinforcement bars and subjected to cyclic loads are particularly sensitive to the bond-slip mechanism.

Existing experimental and numerical studies on the cyclic behaviour of RC structures generally refer to structures with deformed bars. Consequently, the behaviour of RC elements with smooth bars, namely in terms of the bond-slip mechanism, is not fully characterized and understood.

In general terms, the work strategy planned for the development of the thesis consists in three main parts: experimental tests, development and calibration of numerical models, and application of these models in the structural analysis of existing constructions (case-studies).

A series of experimental tests will be performed on RC elements built with smooth reinforcement bars, representative of RC structures built before the 70's, that will be subjected to cyclic loading.

The test campaign has already been initiated with the testing of a RC beam from an ancient building (Santa Joana Museum, Aveiro, Portugal), built with smooth reinforcement bars. The beam was subjected to two loads of equal intensity (symmetric loading), with application point located at each mid-span section, applied in unidirectional cycles of increasing intensity (see Figure 13).

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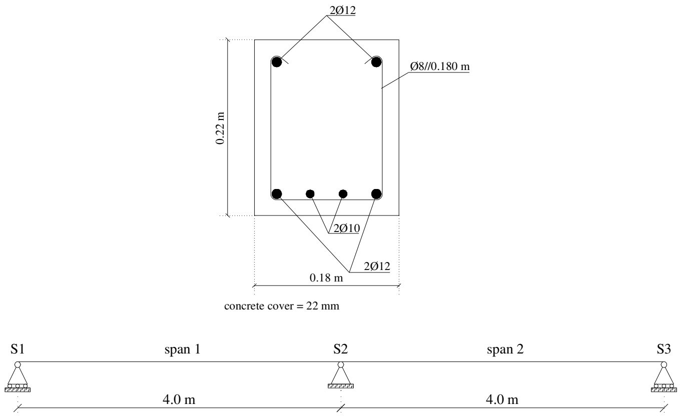

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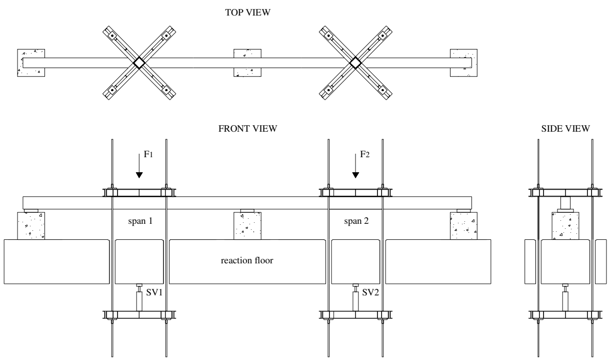

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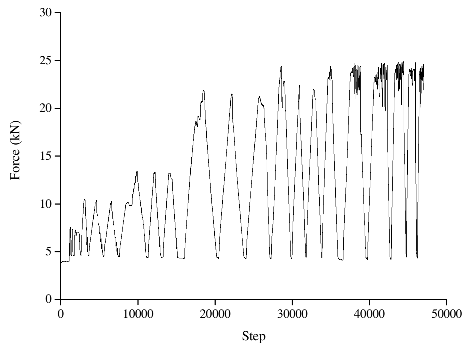

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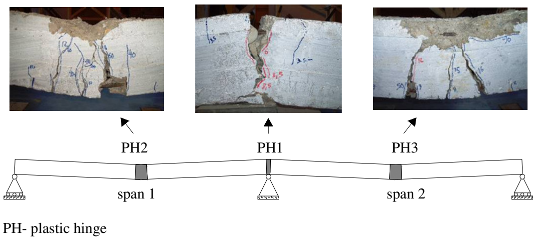

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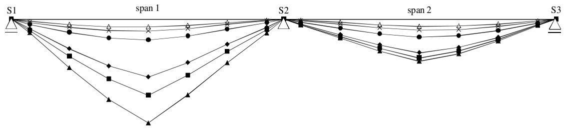

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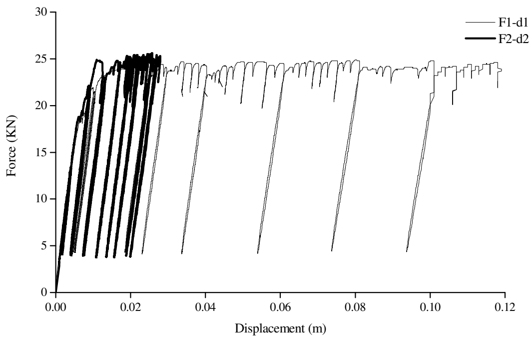

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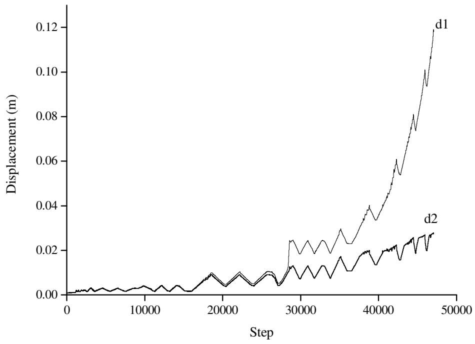
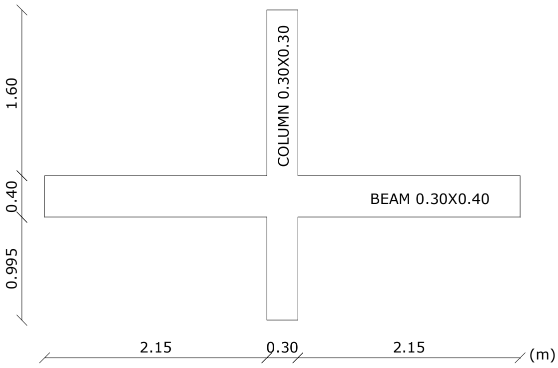


i)

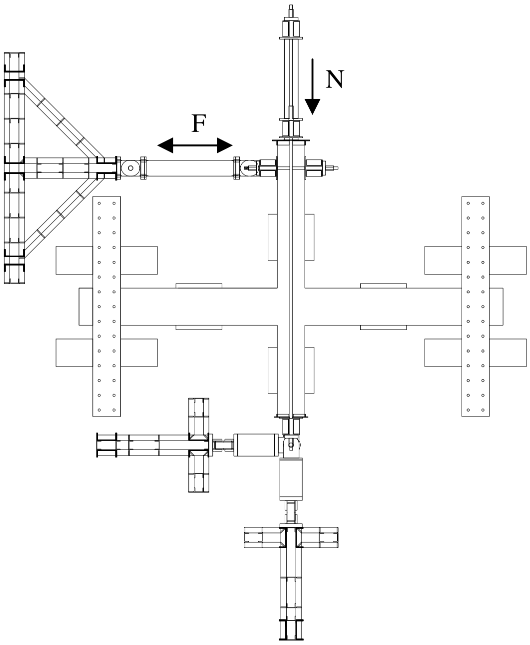
Figure 13. Test of a RC beam: a) beam at the Santa Joana Museum; b) beam dimensions and support conditions; c) test set-up; d) loading history; e) general view of the testing area; f) plastic hinges location; g) deformed shape; h) force-displacement curves; i) vertical displacement evolution at mid-span sections

The test campaign will continue with the testing of a set of RC beam-column joints, built with smooth reinforcement bars according to the old design and construction practices. The joints will be subjected to reversed cyclic loads, with displacement control, to be applied at one of the column extremities, according to the test set-up illustrated in Figure 14-b.



a)

c)



b)

d)

Figure 14. RC beam-column joints: a) general dimensions; b) general test set-up; c) construction of the test samples; d) samples at the laboratory

For reproducing the experimental behaviour of the RC elements, numerical models will be developed using the Open System for Earthquake Engineering Simulation (OpenSees) and calibrated with the experimental results. The effects of the bond-slip mechanism on the structural response of the elements will be considered in the numerical models by using the bond-slip model available in OpenSees. This is expected to result in a better approach between the numerical and the experimental results.

The numerical model of the beam has already been developed and the confrontation between the numerical and the experimental results allows concluding that considering bond-slip leads to a more realistic simulation of the structural behaviour of the beam. The model for predicting the behaviour of the beam-column joints to be tested is under development. After the experimental test, the joints, the experimental results will be used to calibrate the model of the joints.

A second test campaign is planned regarding the beam-column joints. After the first tests, the joints will be repaired and strengthened according of different strengthening techniques, and subjected to new cyclic loading. The confrontation between the experimental and numerical results will allow taking preliminary conclusions about the most efficient strengthening techniques to be used in RC constructions built with smooth reinforcement bars.

In the final stage of the work, the numerical models developed will be used for the vulnerability assessment of existing RC buildings, representative of structures built before the 70’s, so that the reliability of these models can be tested.

5. BIAXIAL BENDING RESPONSE OF REINFORCED CONCRETE COLUMNS

The behaviour of axially loaded reinforced concrete members under biaxial bending moment reversals is generally recognized as a very important topic, for a number of relevant reasons. On the one hand the actual response of RC frame columns to horizontal actions is in general three-dimensional (3D); on the other hand, the 2D features of bending moments histories applied to a given RC column section tends to reduce its actual capacity and to accelerate the strength and stiffness deterioration process during successive load reversals. In addition, the 3D response of frame structures to actual earthquake motions generally does not induce the same type of increased deterioration in beams because they behave essentially in only one direction (vertical), i.e. the potential formation plastic hinges in beams is not aggravated by that fact. This means that both the 2D loading effects in columns and the 3D features of the general structure response positively contribute to inelasticity and damage concentration in the columns rather than in the beams, which is essentially the opposite of present-day design code requirements to avoid collapse of RC frame structures under lateral load reversals (plastic hinges in the beams rather than in the columns).

The interest in the inelastic 3D response of axially loaded members under biaxial bending moment histories is relatively recent and the available experimental results are limited. Possibly, this is partially due to the uncertainty of combining bending moments' histories in the two orthogonal directions that adds considerable complication to the problem. The practical result is that our present-day knowledge of the inelastic behaviour of RC columns under biaxial cyclic moments is very much behind our understanding of the behaviour under 1D cyclic bending with axial load.

The available test results for biaxial bending under constant axial load are not so extensive when compared to those on 1D bending, although they have been delivered over a period of almost 30 years. Contributions can be found for instance from Takizawa and Aoyama (1976), Otani *et al.* (1980), Saatcioglou and Ozcebe (1989), Bousias *et al.* (1992), Kim and Lee (2000), Qiu *et al.* (2002), Tsuno and Park (2004), Nishida and Unjoh (2004), Umemura and Ichinose (2004) and Kawashima *et al.* (2006).

This research develops an experimental program that intends to close the existing lack of experimental results in the literature (Figure 15). The results should increase the knowledge concerning the experimental behaviour of biaxial RC columns and also enable to validate the numerical models proposed for slender columns and to verify the application of the different simplified design methods of columns subjected to biaxial bending, proposed by the Standards.

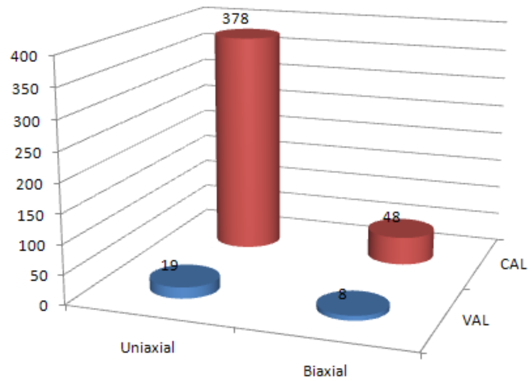


Figure 15. Classification of the cyclic tests available in the literature for rectangular columns according to the testing direction (uniaxial, biaxial) and with the axial force evolution (constant, variable)

In general, most important findings agree that, besides the expected significant influence of axial loads on the hysteretic response of columns, the 2D transversal load cycles are responsible by increased strength and stiffness degradation when compared to the 1D response. In addition, the failure mechanism is found to be very dependent of the loading path and history and strongly affects both the ductile and energy dissipation capacity of the column. On the other hand, there is some experimental evidence that plastic hinge zone lengths tend to be stable at around theoretical values and are not very affected by 2D loading. Due to testing difficulties and because there are still open questions regarding the cyclic behaviour both in 2D bending with constant axial force and in 1D bending with simultaneously varying axial load, very few experimental studies have, as yet, tackled the more general problem of 2D bending with varying axial force.

5.1. Experimental study

The present experimental study is included in a large study promoted by the Laboratory of Earthquake and Structural Engineering (LESE), of FEUP for the experimental study of RC columns (buildings and bridges) under earthquake loadings. In the present is intended to continue a previous experimental campaign on building RC columns. For the first campaign were constructed two specimens with the same geometric characteristics to test under cyclic horizontal loading. The longitudinal reinforcement is composed by six bars with 12mm and the transversal reinforcement have 6mm, 150mm spaced. For the footing were used bars of 16mm diameter. In Figure 16 is presented the specimen detailing for the column and footing.

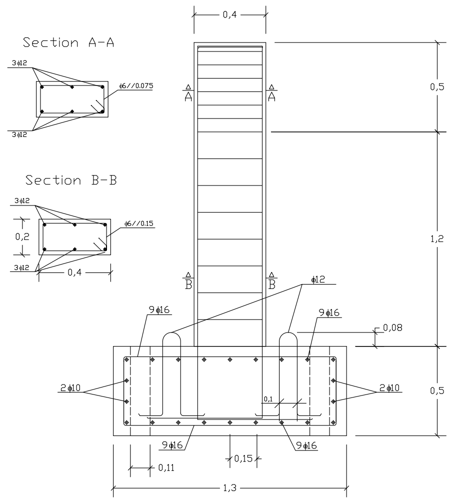
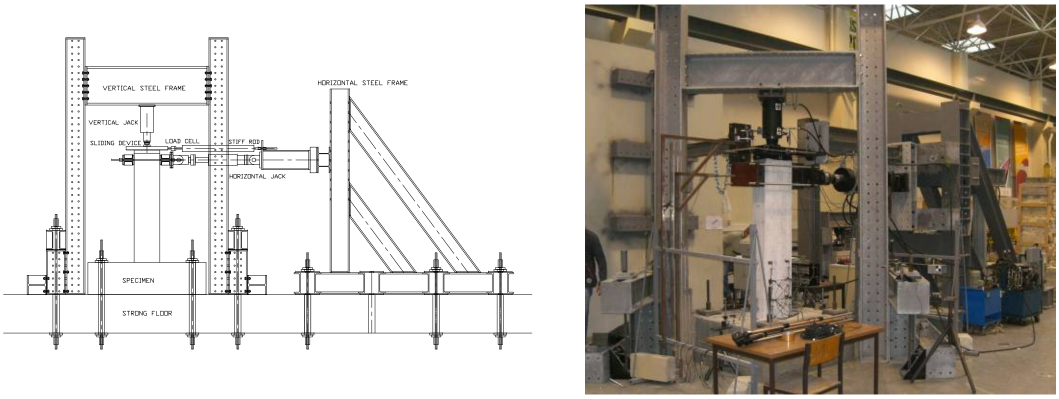


Figure 16. Specimen dimensions and reinforcement details

The two column prototypes are composed by a column with 1.70m high and a heavy square concrete foundation consisting in a block with $1.30x1.30m^2$ in plan and a with 50cm height. This procedure intends avoid sliding and overturning of the specimen during testing.

5.2. Testing setup

In Figure 17 is presented the setup adopted for the experimental campaign. The system includes two horizontal actuators to apply the lateral loads, one with 500kN with +/-150mm stroke and the other with 200kN with +/-100mm stroke and a 700kN actuator to apply the axial load, using two reaction frames (lateral an vertical), and one reaction wall (lateral). The specimens and the reactions frames were fixed to the laboratory strong floor (with prestressed Diwidag steel bars) to avoid sliding and overturning of the specimen during testing and sliding of the reaction frames.



a) b)

Figure 17. Testing setup at LESE laboratory: a) schematic layout; b) general view

In the tests described below, a constant axial load was applied while the lateral loading was cycled under displacement controlled conditions. Since the axial load remains in the same position while the specimen deflects, a special sliding device is used in order to minimize spurious friction effects. The hydraulic system of the vertical actuator was designed to keep the oil pressure constant, thus ensuring a constant axial force during the tests. The horizontal actuators control and the data acquisition are both performed through a PXI controller system from National Instruments together with specifically home developed control routines based on the LabVIEW software platform. Data acquisition and signal conditioning cards provide direct readings from load cells, LVDTs (Linear Variable Displacement Transformer) and other types of amplified analogical or digital sensors.

The instrumentation schemes adopted a along the column height using the LVDT layout shown in Figure 18, allows to measure the total deformation and after the test based on a simplified approach it can be obtained the flexural and shear components of the specimen deformation.

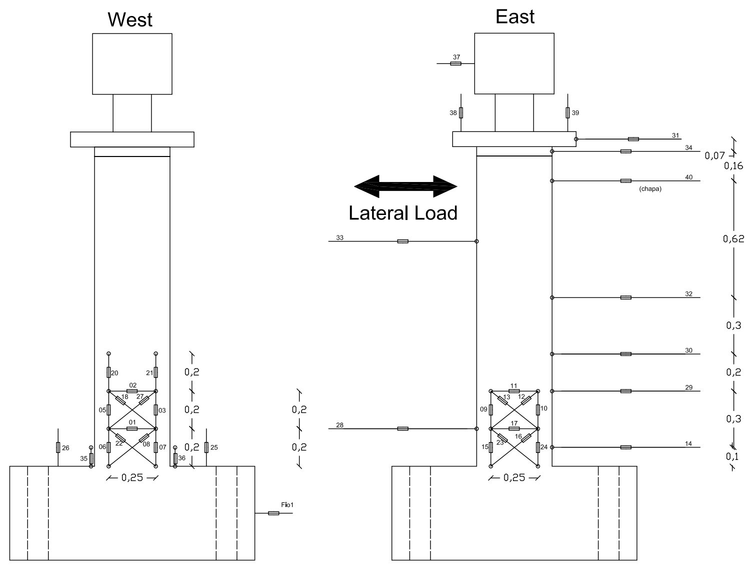


Figure 18. Instrumentation scheme

5.3. Experimental results

The most relevant results obtained from the four tests (specimens PB01-N1, PB02-N1), are reported hereafter for each test independently presenting the damage pattern and the results in terms of actuator force *vs.* top displacement, dissipated energy, variation of the axial force during the test and finally the out-of-plane displacement measured.

5.3.1. PB01-N1 Specimen

In Figure 19 is presented the damage pattern observed during the test and in Figures 20 and 21 are presented the global results in terms of actuator force *vs.* top displacement and dissipated energy.

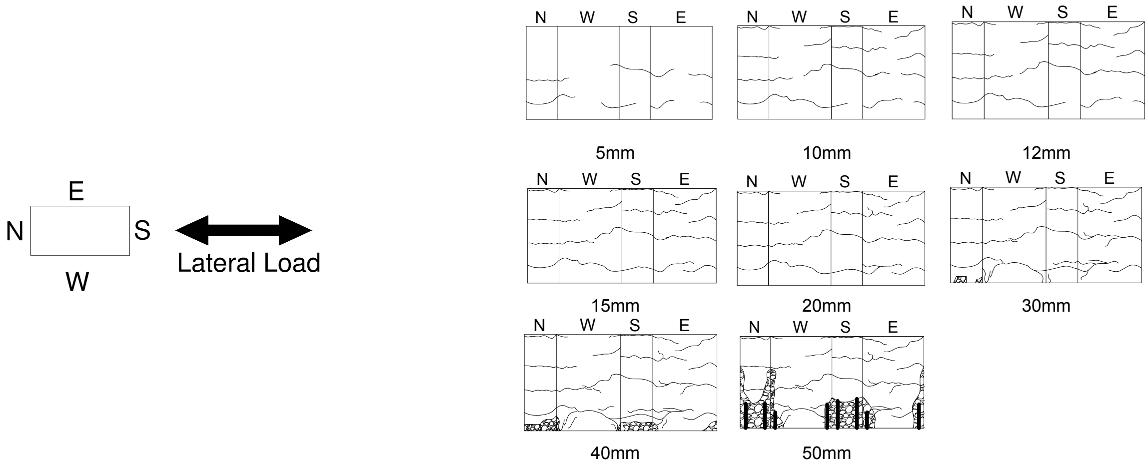


Figure 19. Damage pattern in each 3rd cycle for PB01-N1

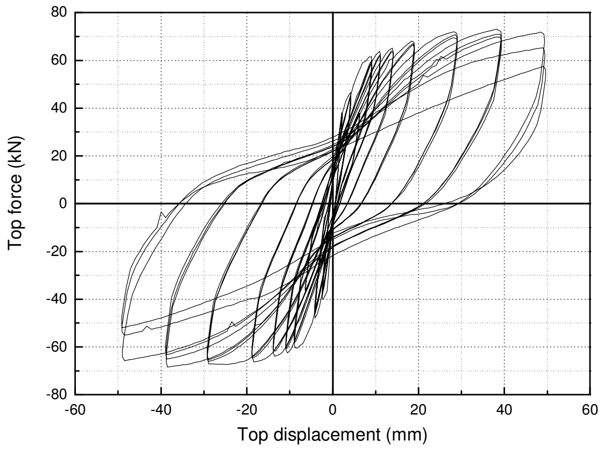


Figure 20. Force *vs.* top displacement

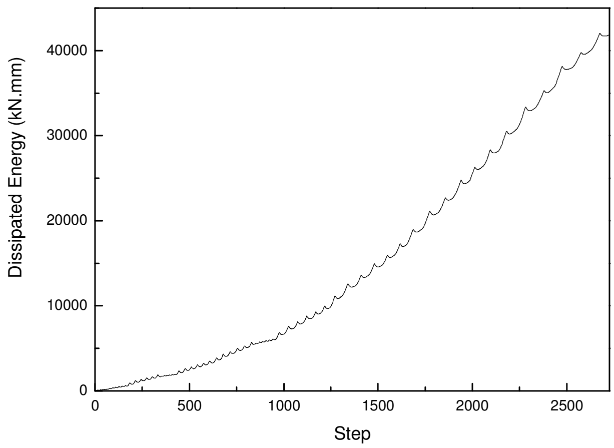


Figure 21. Dissipated energy

5.3.2. PB02-N1 specimen

In Figure 22 is presented the damage pattern observed during the test and in Figures 23 and 24 are presented the global results in terms of actuator force *vs.* top displacement and dissipated energy.

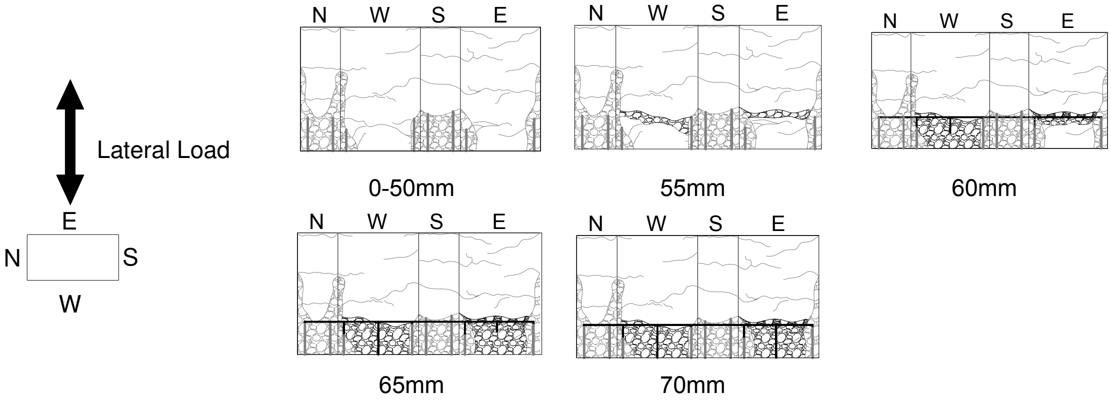


Figure 22. Damage pattern in each 3rd cycle for PB02-N1

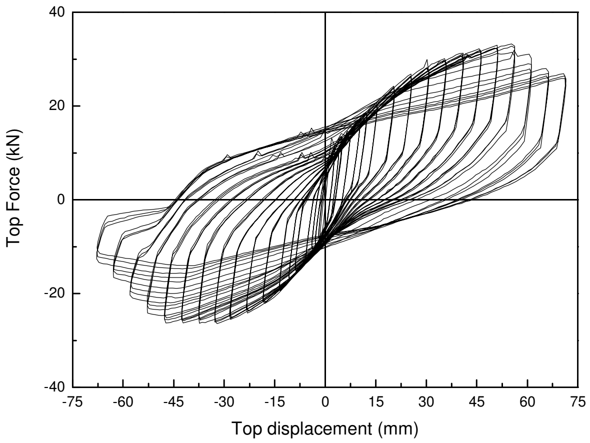


Figure 23. Force *vs.* top displacement

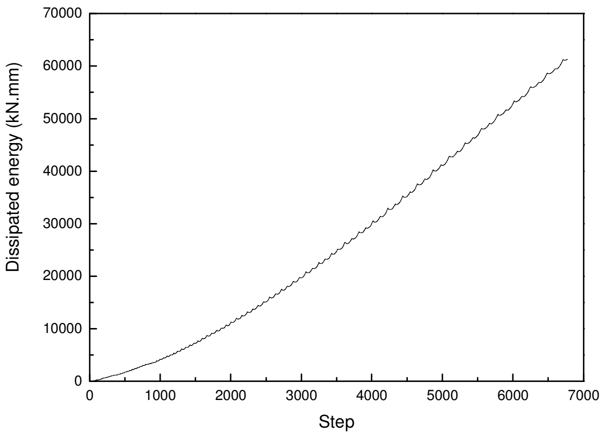


Figure 24. Dissipated energy

6. CONCLUSIONS

The pseudo-dynamic tests performed on the 4-storey full-scale RC models have shown that the vulnerability of existing reinforced concrete structures constitute a high risk source for human life. Furthermore, it was demonstrated that retrofitting solutions adequately selected, designed an implemented can reduce substantially that risk to levels currently considered in modern design.

The numerical results obtained with the proposed non-linear displacement-based model are in good agreement with the experimental ones, even for the irregular structure. The proposed optimization methodology deal with non-linear objective functions and allow to impose constrains on the design variables (strength, stiffness or damping) and on any other response variable, depending on the design variables, such as inter-storey drift, top displacement, etc., generating the optimum strengthening storey distribution, for one or multiple performance objectives. These simplified models can be useful design tools, as a preliminary step, in the seismic vulnerability assessment and global structural strengthening decision, which could allow for parametric studies and rapid screening of existing building classes.

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